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# BED SHEAR STRESS COEFFICIENT WITHIN THE SURF ZONE

Carlos Severino Veitia Garcia

# NAVAL POSTGRADUATE SCHOOL

Monterey, California

# THESIS

## BED SHEAR STRESS COEFFICIENT WITHIN THE SURF ZONE

by

Carlos Severino Veitia Garcia

September 1977

Thesis Advisor: E. B. Thornton

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sinusoidal wave solution. Field measurements of longshore current and waves within the surf zone are used to calculate the bed shear stress coefficient. The data consist of 647 data points selected from LEO program and 62 data points from Ingle (1966) observations, all taken along the Southern California coast. Frequency distributions and statistics are calculated for the bed shear stress coefficient. A mean bed shear stress coefficient to two significant decimal places is found to be 0.01.

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# Bed Shear Stress Coefficient
# within the Surf Zone

by

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Commander, Venezuelan Navy
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Submitted in partial fulfillment of the
requirements for the degree of

MASTER OF SCIENCE IN OCEANOGRAPHY

from the

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# ABSTRACT

An analytical formulation of the bed shear stress coefficient inside the surf zone is derived using the concept of radiation stress. A truncated Rayleigh p.d.f. is used to describe the wave field inside the surf zone and provides the input to calculate the variation of wave energy and longshore current as a function of wave height, water depth and distance to shore. The wave set-up is approximated using a sinusoidal wave solution. Field measurements of longshore current and waves within the surf zone are used to calculate the bed shear stress coefficient. The data consist of 647 data points selected from LEO program and 62 data points from Ingle (1966) observations, all taken along the Southern California coast. Frequency distributions and statistics are calculated for the bed shear stress coefficient. A mean bed shear stress coefficient to two significant decimal places is found to be 0.01.

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# LIST OF SYMBOLS

$C$ - Wave celerity

$C_b$ - Wave celerity at breaking

$C_f$ - Bed shear stress coefficient

$C_g$ - Speed of wave energy propagation

$D$ - $\bar{\eta} + h$, total depth of water

$E$ - Energy density

$g$ - Acceleration due to gravity

$h$ - Local depth below still water level

$h_b$ - Depth below still water level at breaking

$\bar{H}$ - Local mean wave height

$H_b$ - Significant wave height at breaking

$H_s$ - Local significant breaker wave height within surf zone

$i$ - Index corresponding to horizontal coordinate in X-direction

$j$ - Index corresponding to horizontal coordinate in Y-direction

$S$ - Bottom slope

$S_{ij}$ - Excess of momentum flux tensor (radiation stress)

$Uw$ - Water particle velocity due to wave motion

$V$ - Mean velocity component parallel to the beach

$x$ - Horizontal coordinate perpendicular to the beach

$x_b$ - Width of the surf zone

$y$ - Horizontal coordinate parallel to the beach

$\alpha$ - Incident wave angle

$\alpha_b$ - Incident wave angle at breaking

$\bar{\eta}$ - Mean water surface elevation

$\bar{\eta}_b$ - Mean water surface elevation at breaking

$\gamma$ - Ratio breaking wave height to depth of water at breaking

$\rho$ - Water density

$\tau_b$ - Bottom shear stress

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# I. INTRODUCTION

## A. STATEMENT OF THE PROBLEM

It is known that when sea waves or swell approach a straight coastline at an oblique angle a mean current is generated parallel to the shoreline, see Figure 1. Such longshore currents are of prime importance for both coastal engineering and for aiding in the strategic planning of Naval inshore warfare operations.

An accepted theory of longshore currents on plane beaches is developed in terms of the momentum flux due to the waves directed down coast being balanced by the shear stress associated with the mean flow. The formulation of the bed shear stress requires the specification of a bed shear stress coefficient. The purpose of this thesis is the determination of the bed shear stress coefficient to be used in the longshore current formulas.

The study will also help in the analysis of sediment transport. The shear stress does work on the bottom in moving sediments. Several authors have formulated sediment transport in terms of the bed shear stress which in turn requires an appropriate bed shear stress coefficient.

## B. HISTORICAL REVIEW

Inman and Quinn (1952), using the momentum approach for the prediction of longshore current by Putnam, Munk and

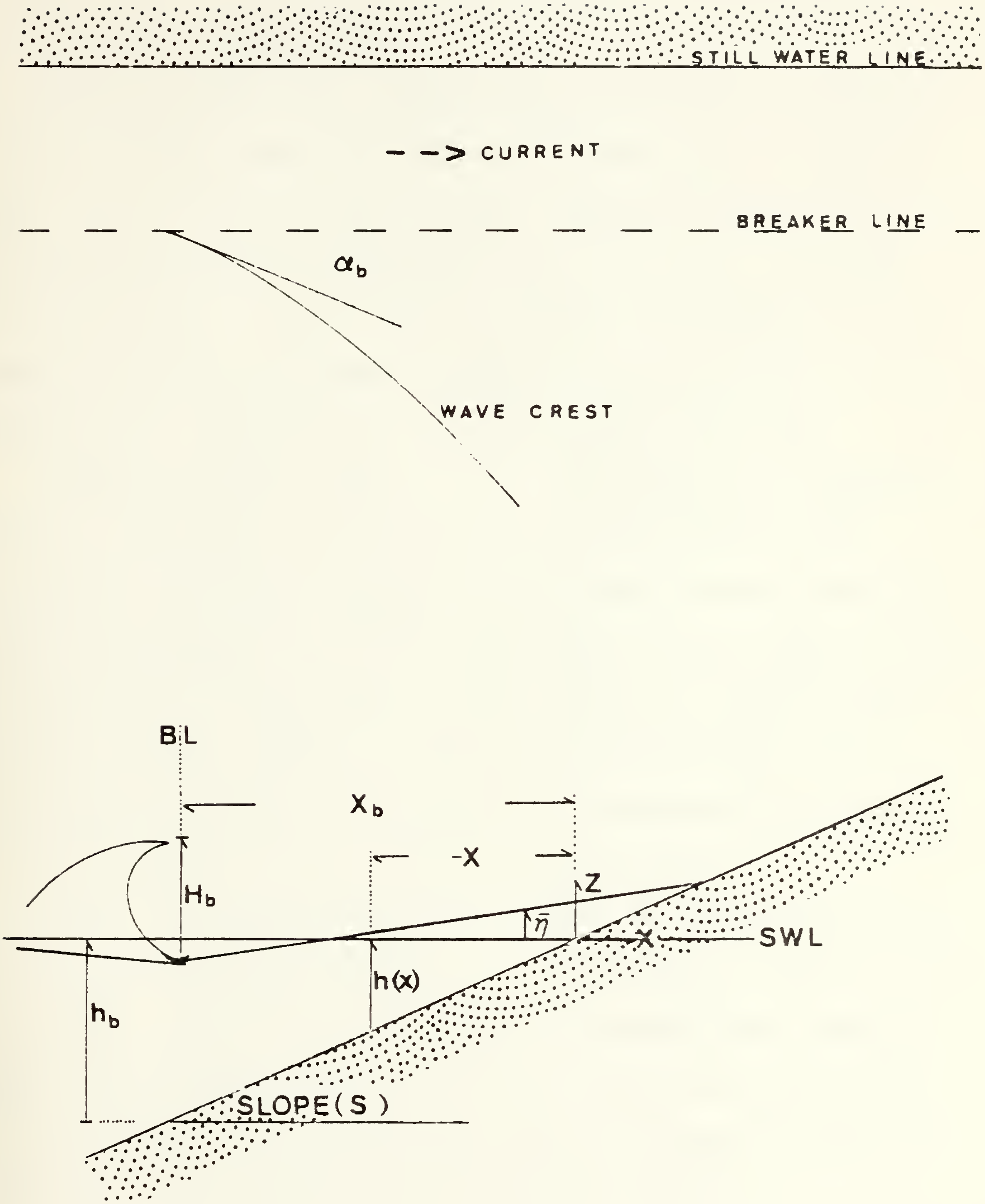


Figure 1. Definition of Longshore Current Variables.

Taylor (1949), showed that in order to fit theory with observations, the bed shear stress coefficient must be permitted to vary with the longshore velocity over a wide range of 3 1/2 orders of magnitude.

Bretschneider (1954) found that the spectral limitations of wave growth, under the action of steady wind in shallow water with a typical sandy bottom, suggested a value for the friction coefficient of between 0.01 and 0.02. Also, he found that the observed damping of swell propagating over a smooth, leveled, impermeable sea bed was consistent with a value of the coefficient of between 0.034 and 0.097.

Longuet-Higgins (1970), using the concept of radiation stress, developed a relationship for prediction of the theoretical maximum longshore current just inside the breaking and proposed a friction coefficient of the order of 0.01. He concluded, on the basis of the finding of Bretschneider (1954), Prandtl (1952) and Nikuradse's experiment with roughened pipes, that is was not "...unreasonable to expect a friction coefficient of the order of 0.01."

Table I was taken from Sonu (1975); it summarizes some values of the friction coefficient proposed by various investigators. The values reported were obtained from measurements outside the surf zone or from laboratory experiments. It can be seen from this table that the range of values is relatively wide and the test conditions varied.

TABLE I. Bottom Friction Coefficients Proposed by Various Investigators

| Friction Coefficient | Wave Height (meters) | Wave Period (sec) | Test Conditions | Authors |
|---|---|---|---|---|
| 0.01 | Arbitrary | Arbitrary | Shallow water steady state wave generation | Bretschneider (1954,a) |
| 0.030-0.089 | 0.23-0.51 | 2.88-3.96 | Gulf of Mexico; depths 3.4-5.2 m slope 0.00035-0.00-41 | Bretschneider (1954,b) |
| 0.030-0.040 | 2 | 10 | Niigata, Japan; depths 2.25-2.75 m slope 0.018 | Kishi (1954) |
| 0.01 | 5.4* | 8.4* | Oscillating water channel, turbulent boundary layer | Jonsson (1966) |
| 0.01 -0.40 | 0.002-0.100 | 0.88-2.58 | Wave flume, laminar boundary layer | Iwagaki and Tsuchiya(1966) |
| 0.03 -0.18 | 1.77-2.47 | 9.1-15.5 | Hiyshizu, Japan; depths 13-10 m slope 0.0060 | Iwagaki and Kakinuma(1966) |
| 0.03 -0.15 | 1.05-1.60 | 7.4-12.5 | Takahama, Japan; depths 10-7 m slope 0.0057 | Iwagaki and Kakinuma(1966) |
| 0.09 -0.50 | $(UT/2\pi\eta) \simeq 4 \sim 20$<br>U: nearbottom velocity<br>$\eta$: ripple height | | Wave flume study; derived from energy dissipation in sand ripple vortices | Tunstall and Inman (1975) |

Note: *Equivalent values at 10 m depth

## C. OBJECTIVES OF THE STUDY

The bed shear stress coefficients previously determined are based on a very limited set of field data or on laboratory studies which used as a model simple sinusoidal waves which are not typical of the randomness found in nature. The objective of this study is to analyze existing sets of field observations obtained in the surf zone and by using the best available theory attempt to determine a reasonable value of the bed shear stress coefficient. For this purpose a fairly large data set obtained for the Channel Island Littoral Environment Observations (LEO) Program was used as well as a set of observations by Ingle (1966) taken at various locations along the Southern California coast. It is expected that the data obtained and the theory applied will ultimately contribute to the establishment of a reasonable value of the bed shear stress coefficient and to a more accurate prediction of the longshore current velocity across the surf zone.

# II. THEORY

## A. INTRODUCTION

Several models have been proposed for the distribution of the longshore current velocity across the surf zone on a plane sloping beach. The solution,which uses pure sinusoids to describe the waves and no lateral shear stress, gives a velocity distribution which is triangular shaped with both a peak velocity and a discontinuity at the breaker point [(Bowen, 1969), Thornton (1969) and Longuet-Higgins (1970)]. This is unreasonable since there are no discontinuities in nature. A second model including lateral shear stress tends to smooth out the discontinuity at the breaker line and produces a smoother velocity distribution with the maximum velocity occurring closer to shore. However, no criterion to predict an optimum lateral shear stress coefficient is as yet available. This introduces an added complexity to the problem.

A random-sea model developed by Collins (1972) circumvents the difficulty of the lateral shear stress coefficients and allows the statistical input of the sea state as described by a Rayleigh distribution. Figure 2 compares the velocity distribution resulting from the various models: the non lateral stress model, the lateral shear stress model for a coefficient equal to 0.4 (Longuet-Higgins, 1970), and the

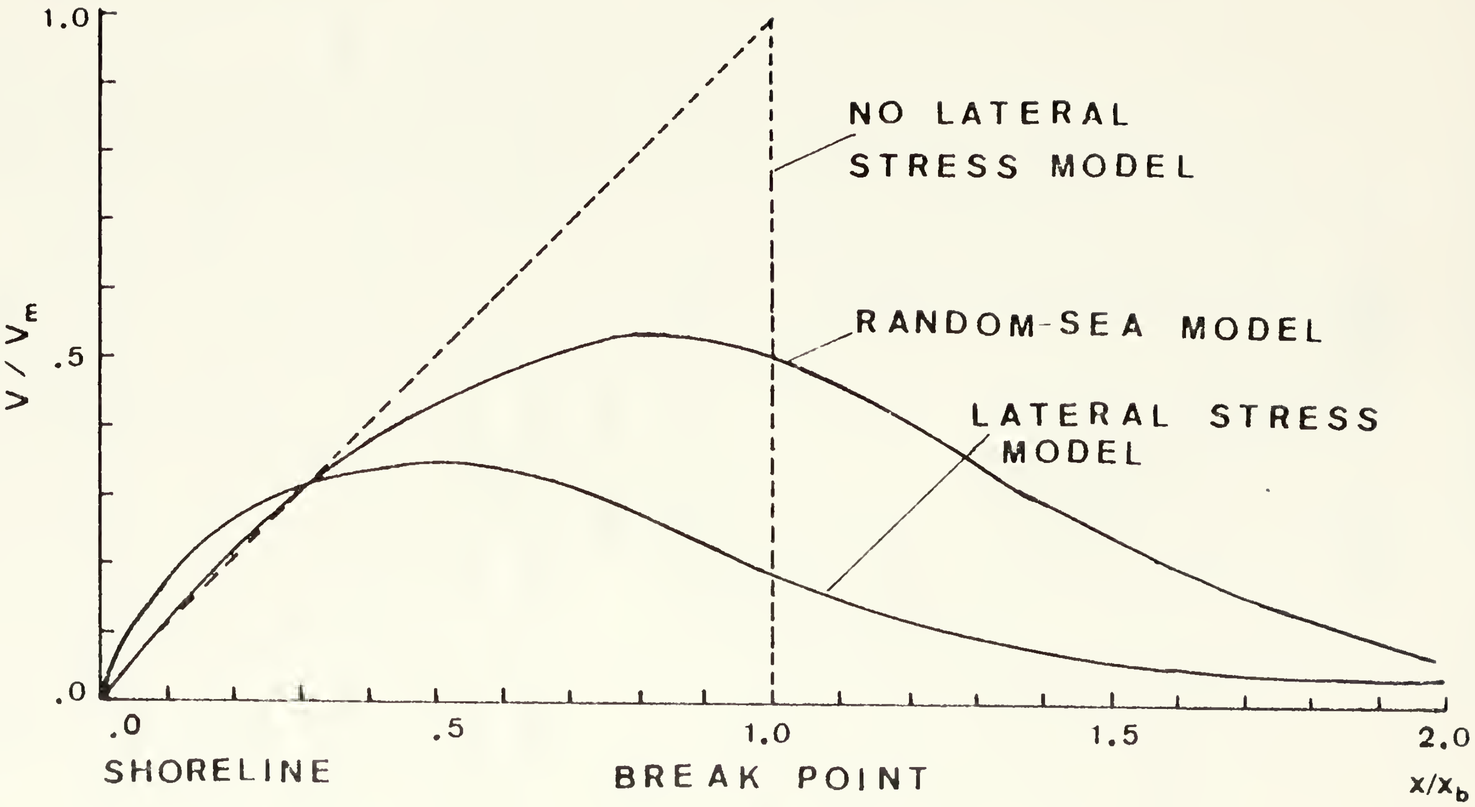


Figure 2. Comparison of Longshore Current Models.

random-sea model. The velocities are referenced to the peak velocity, Vm, of the non lateral shear stress model, which according to Longuet-Higgins (1970) is given by

$$Vm = \frac{5\pi g}{16C_f} H_b \; S \; \frac{\sin\alpha_b}{C_b}$$

and V is the longshore component of the mean current velocity.

Bowen (1969), Thornton (1969) and Longuet-Higgins (1970) attributed the generation of longshore currents, due to an oblique wave approach, to the longshore component of the momentum flux (radiation stress) of the water waves. The calculation of the wave-induced longshore current velocities and changes in mean water level requires the specification of the radiation stresses as a function of the location and wave properties in the nearshore region.

## B. WAVE SET-UP INSIDE THE SURF ZONE

As waves approach the coast and shoal there is a change in the momentum flux of the waves which is balanced by a change in the mean water level. Outside the surf zone there is a set-down while inside the surf zone, after breaking, there is a set-up or superelevation of the water level. The wave set-up is important because both the local wave height and speed are functions of the total local water depth which is unknown. The change in mean water level required to balance the excess momentum flux of the waves must be determined first. A convenient form of the x-component (shoreward) of the momentum flux equation integrated over

depth and averaged in time, derived by Longuet-Higgins and Stewart (1962), for describing the wave set-up is given by

$$\frac{\partial S_{xx}}{\partial x} + \rho g(h+\bar{\eta}) \frac{\partial \bar{\eta}}{\partial x} = 0 \qquad (1)$$

which says that the change of excess momentum flux due to wave action ("radiation stress") is balanced by a change in the mean water level. It is assumed in the derivation that the net local mass flux perpendicular to shore is zero so that there is no contribution from the mean motion to the momentum flux perpendicular to the shore, and that the mean stresses are negligible.

Inside the surf zone it is assumed that the radiation stress tensor can be expressed in terms of the energy and wave speed in the same form as in shallow water. This implies that the breaking waves are of the spilling type and that even under breaking waves the water particle motion retains much of its organized character as described by linear wave theory. Using the shallow water approximation, that the group velocity equals the phase velocity and that the angle of incidence equals zero, the radiation stress term reduces to

$$S_{xx} = \frac{3}{16} \rho g \gamma^2 (h+\bar{\eta})^2 \ . \qquad (2)$$

Substituting (2) into (1) and integrating gives the mean water elevation of the form

$$\bar{\eta} = K(h_b - h) + \bar{\eta}_b \ , \qquad (3)$$

where

$$K = \frac{1}{1 + \frac{1}{3/8\,\gamma^2}} \,, \qquad (4)$$

and the mean water elevation at breaking is given by

$$\bar{\eta}_b = - \frac{\gamma H_b}{16} \,. \qquad (5)$$

$H_b$ is a single breaker height which corresponds to the significant wave height observed at the breaking position. For this study, it was assumed that the waves followed the breaking index,

$$\gamma = \frac{H_b}{h_b} = 0.78 \,,$$

derived from a modified solitary wave theory by Munk (1949). The total local depth of water, D, is obtained by combining the local mean water super elevation and the local depth,

$$D = \bar{\eta} + h \,. \qquad (6)$$

A sinusoidal description of the waves was used to solve for the wave set-up in order to get a closed form analytical solution and to circumvent the difficult numerical solution of equation (1) required by the random-sea model. Collins (1972) compared the sinusoidal solution to the random sea solution as shown in Figure 3. The effects of the random-sea model is to smooth the wave's set-up curve. The magnitude of the difference between the two solutions is very small but

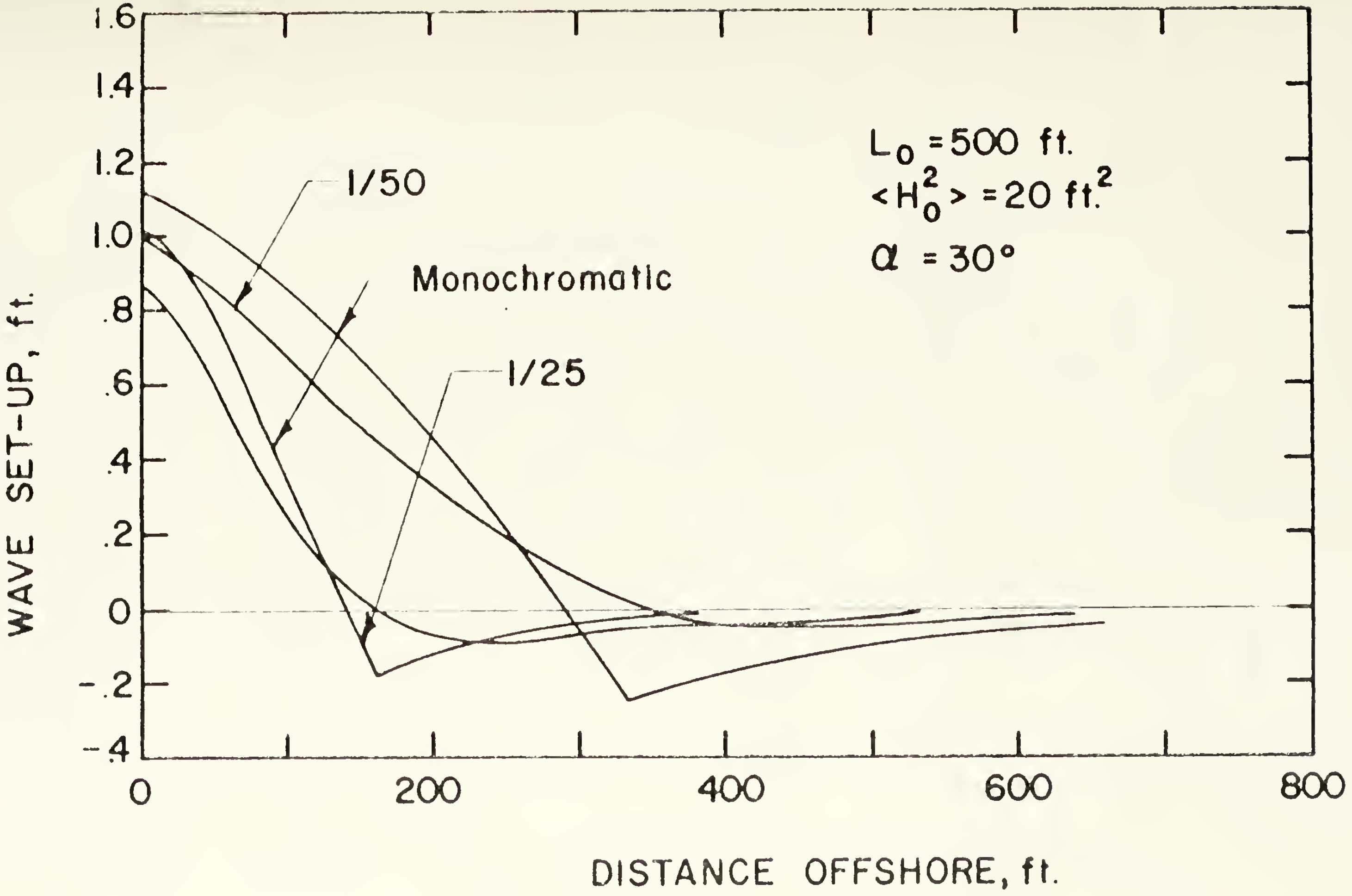


Illustration of wave set-up on offshore slopes of 1:25 and 1:50, broken lines indicate results for periodic waves having the same energy content.

Figure 3. Comparison of Wave Set-up Solutions.

the percent difference can be large as the depth of water approaches zero. The total depth of water is important in prescribing the breaker point or the limits of integration on the Rayleigh distribution for the random-sea model. In the calculation of longshore currents using the random sea model it is the area under the Rayleigh distribution that is used so that small errors on the limits generally cause only even smaller errors in the area. Hence, it is felt using sinusoidal wave descriptions to calculate wave set-up is a reasonable approximation.

## C. WAVE FIELD INSIDE THE SURF ZONE

A description of the wave field is required in the longshore current calculation because knowledge of it is needed for specifying the horizontal water particle velocities and for determining the longshore component of the radiation stresses in an irregular wave field. Inside the surf zone the waves are unstable and the fluid motion loses some of its ordered character; but Thornton (1976) points out that most of the water particle motion in the body of the fluid is coherent with the surface and can be considered wave-induced and not turbulent, particularly for spilling type breakers.

In this study, a truncated Rayleigh distribution as shown in Figure 4 is used to give a statistical description of the wave field as described by Collins (1972) and Battjes (1974). The basic assumption is that at each depth a limiting breaker

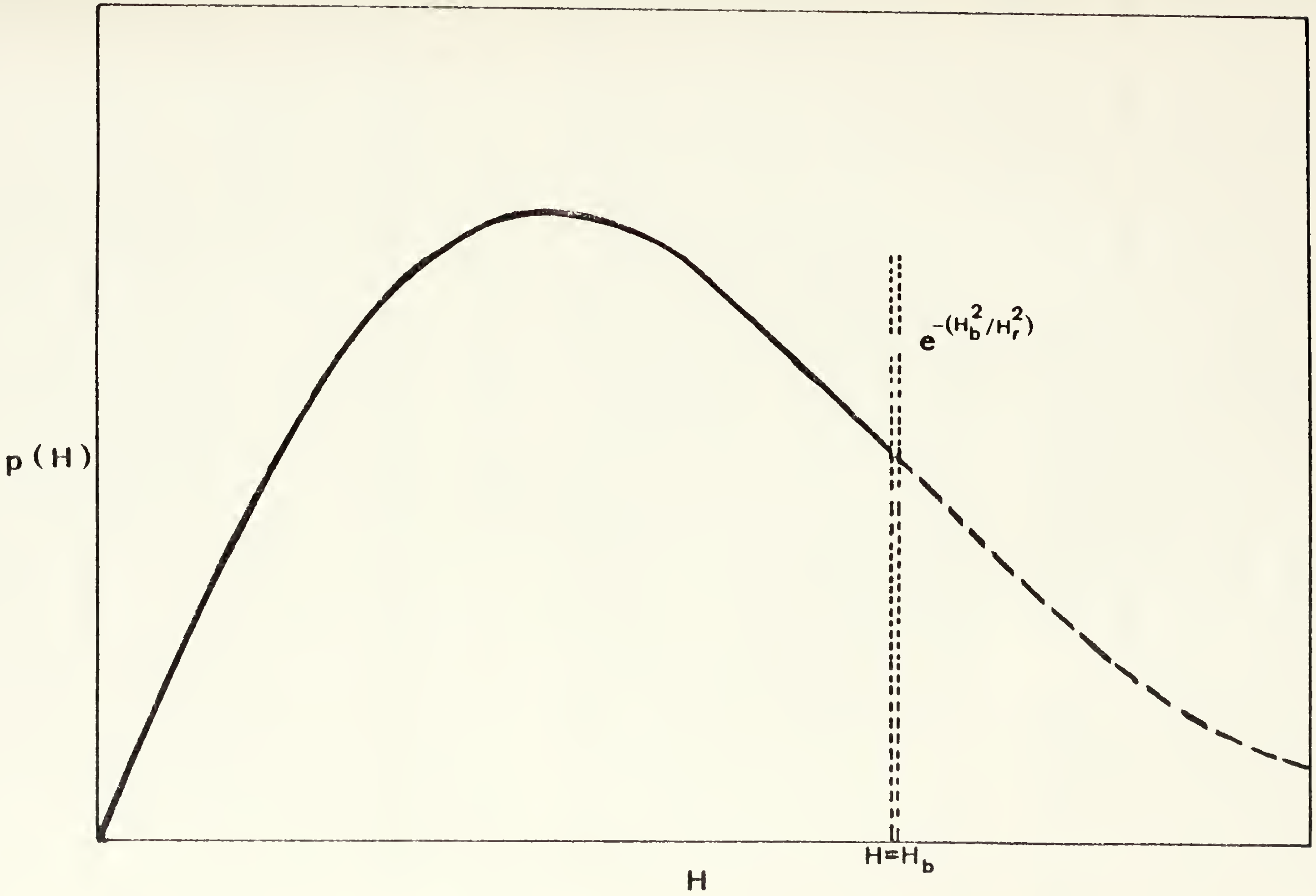


Figure 4. Truncated Rayleigh p.d.f.

height can be defined which cannot be exceeded by the individual waves of the random field, and that those wave heights which in the absence of breaking would exceed the breaker height are reduced by breaking to the value of the local breaker height. That is, the energy corresponding to the height in excess of the local breaker height is assumed to be dissipated. The limiting breaker height decreases as depth decreases.

In describing the Rayleigh distribution, a fictitious, or reference, local energy per unit area, denoted as $E_r$, is defined. The reference energy density refers to that energy density that would exist if breaking had not occurred nad accounts for shoaling and refraction transformation. Battjes (1974) also defines reference wave heights $H_r$ and their mean square value $\overline{H_r^2}$ which is related to $E_r$ according to

$$E_r = \frac{1}{8} \rho g \overline{H_r^2} . \tag{7}$$

The reference wave heights are assumed to be Rayleigh distributed.

The Rayleigh wave height distribution is clipped at $H = H_s$ in accordance with the assumption that the height of a breaking wave equals the local breaker height, $H_s$, in order to obtain an approximation to the actual wave height distribution. Then, the mean energy per unit area at a fixed point, taking account of breaking, is calculated from

$$E = \frac{1}{8} \rho g \overline{H^2} . \tag{8}$$

The variance is calculated from the pdf of H,

$$\overline{H^2} = \int_0^\infty H^2\, p(H)\, dH \ , \tag{9}$$

where

$$p(H) = \int_0^{H_s} d[1-\exp(-H^2/\overline{H_r^{\,2}})] + H_s^{\,2} \exp(-H_s^{\,2}/\overline{H_r^{\,2}}) \ , \tag{10}$$

$$\overline{H^2} = \int_0^{H_s} H^2 d[1-\exp(-H^2/\overline{H_r^{\,2}})] + H_s^{\,2} \exp(-H_s^{\,2}/\overline{H_r^{\,2}}) \ , \tag{11}$$

$$\overline{H^2} = [1-\exp(-H_s^{\,2}/\overline{H_r^{\,2}})]\ \overline{H_r^{\,2}} \ . \tag{12}$$

The clipped Rayleigh distribution implies that all waves from $H_s$ to infinity that were previously larger than $H_s$ now are reduced to the same height, $H_s$. Therefore, the total probability (percent) of waves having the height $H_s$ is given by

$$\int_{H_s}^\infty p(H)\, dH \ .$$

The contribution to the variance is given by

$$H_s^2 \int_{H_s}^{\infty} p(H)\, dH = H_s^2 \exp(-\frac{H_s^2}{H_r^2}) ,$$

which is the term on the right of equation (11). $H_s$ is the local breaker height, which inside the surf zone is assumed to be given by

$$H_s = \gamma D . \qquad (13)$$

The local mean wave height $\bar{H}$ can be expressed in a similar manner in terms of $H_s$ and $H_r$ by means of the clipped Rayleigh distribution,

$$\bar{H} = \int_{0}^{H_s} H d[1-\exp(-H^2/\overline{H_r^2})] + H_s \exp(-H_s^2/\overline{H_r^2}) \qquad (14)$$

where again the term on the right represents the percent of waves greater than $H_s$ in the original distribution which now have the height $H_s$. Integrating (14) gives

$$\bar{H} = \frac{\sqrt{\pi}}{2} H_{r_{rms}} \operatorname{erf}(H_s/H_{r_{rms}}) \qquad (15)$$

in which

$$H_{r_{rms}} = (\overline{H_r^2})^{1/2}$$

and the error function being defined as

$$\mathrm{erf}(p) = \frac{2}{\sqrt{\pi}} \int_{o}^{p} \exp(-t^2)dt \ .$$

The error function was calculated using the rational approximation of Abramowitz and Stegun (1965),

$$\mathrm{erf}(p) = 1-[(a_1 t + a_2 t^2 + a_3 t^3) \exp(-p^2)] + \varepsilon(p)$$

where

$t = 1/(1+zp)$, $a_1 = .3480242$,

$z = .47047$, $a_2 = .0958798$,

$p = H_s/(\overline{H_r^{\ 2}})^{1/2}$, $a_3 = .7478556$.

The largest error using this approximation is

$$\varepsilon(p) \leq 2.5 \times 10^{-5} \ .$$

In the observations used for comparison with the theory, the breaker height is measured visually. It is assumed that an observer visually measures the significant breaking wave height defined as the average of the highest one-third fraction of the wave heights. The difficulty in applying this definition to the present problem is that the definition applies to a point measurement or a statistically homogeneous (spatial) wave field and in this problem the waves are defined as varying spatially as they shoal shoreward.

In order to define the significant wave height for a spatially varying (nonhomogeneous) wave field, it is assumed that the observer measures waves when spatially one-third of the waves have broken; hence, the reference wave height can be specified from the clipped Rayleigh distribution

$$\exp\left(-H_b^2/\overline{H_r^2}\right) = 0.333$$

and $\overline{H_r^2} = -H_b^2/\mathrm{Ln}(0.333)$ . (16)

## D. LONGSHORE CURRENT VELOCITY

The derivation of the longshore current starts with the y-momentum flux equation

$$\frac{\partial S_{xy}}{\partial x} + \tau_b - \tau_\ell = 0 \ . \qquad (17)$$

Where the lateral shear stress, $\tau_\ell$, is neglected (17) reduces to

$$\frac{\partial S_{xy}}{\partial x} + \tau_b = 0 \ , \qquad (18)$$

which says that the change of y-component (longshore) momentum flux due to waves in the x-direction is balanced by the bottom stress, $\tau_b$, in the y-direction. Assuming that the amplitude of the wave motion $|Uw|$ is much greater than the mean current velocity, V, then (Thornton, 1969)

$$\tau_b = \rho \, C_f \, |Uw| \, V \ . \qquad (19)$$

The excess of momentum flux of the waves, or "radiation stress" component is given by

$$S_{xy} = E \sin\alpha \cos\alpha \frac{C_g}{C} ,$$

which inside the surf zone reduces to

$$S_{xy} = E \sin\alpha \cos\alpha \tag{20}$$

under the assumption that $C_g = C$ for shallow water. Combining equation (8) and (12) gives the mean energy per unit area as

$$E = \frac{1}{8} \rho g \overline{H_r^2}[1-\exp(-H_s^2/\overline{H_r^2})] \tag{21}$$

and

$$S_{xy} = \frac{1}{8} \rho g \overline{H_r^2}[1-\exp(-H_s^2/\overline{H_r^2})] \sin\alpha \cos\alpha . \tag{22}$$

The variables $H_s$ and $\alpha$ can be expressed as differentiable functions of x (distance from shore). Recalling that it was assumed inside the surf zone

$$H_s = \gamma D ,$$

where the total depth is the sum of the still water depth plus the set-up

$$D = \bar{\eta} + h ,$$

from equations (3), (5), (6), and (13) $H_s$ can be expressed as

$$H_s = H_b(K - \frac{\gamma^2}{16}) + h(x)\ (\gamma - \gamma K)\ . \qquad (23)$$

Application of Snell's law allows the local breaker angle, $\alpha$, to be expressed in terms of the known breaking angle $\alpha_b$ and the breaking celerity $C_b$,

$$\sin\alpha = \frac{C}{C_b} \sin \alpha_b\ . \qquad (24)$$

Using the shallow water approximation for wave speed

$$C = (gh)^{1/2}$$

and

$$C_b = (g\ \frac{H_b}{\gamma})^{1/2}\ ,$$

then

$$\sin\alpha = (h\ \frac{\gamma}{H_b})^{1/2}\ \sin\alpha_b\ . \qquad (25)$$

Hence, from equations (23) and (25) it is seen that both $H_s$ and $\alpha$ are now expressed as functions of h, which in turn is a function of x.

The bed shear stress coefficient is determined by combining equation (18) and (19),

$$C_f = \frac{\partial S_{xy}}{\partial x}\ \frac{1}{\rho|Uw|V}\ . \qquad (26)$$

The mean horizontal water particle velocity amplitude is expressed using linear theory (Battjes, 1974),

$$|Uw| = \frac{\bar{H}}{\pi}\frac{C}{D} , \qquad (27)$$

where $\bar{H}$ is given by equation (15).

The change in the radiation stress is given by

$$\frac{\partial S_{xy}}{\partial x} = \frac{E}{2}\left(\frac{\gamma}{H_b}\right)^{1/2} \sin\alpha_b\left(h-\frac{\gamma h^2\sin^2\alpha_b}{H_b}\right)^{-1/2} S\left(1-\frac{2\gamma h\sin^2\alpha_b}{H_b}\right) +$$

$$+ \frac{\sin\alpha\ \cos\alpha}{4}\,\rho g S(\gamma-\gamma K)H_s \exp(-H_s^{\,2}/\overline{H_r^{\,2}}) , \qquad (28)$$

where

$$E = \frac{1}{8}\,\rho g\ \overline{H_r^{\,2}}[1-\exp(-H_s^{\,2}/\overline{H_r^{\,2}})]$$

and

$$\sin\alpha\ \cos\alpha = \left(\frac{\gamma}{H_b}\right)^{1/2} \sin\alpha_b\left(h - \frac{\gamma h^2\sin^2\alpha_b}{H_b}\right)^{1/2} .$$

# III. DATA

## A. LEO DATA

Data from the Littoral Environment Observation (LEO) program established by the Coastal Engineering Research Center was used in this study. In the LEO program, nearly simultaneous observations of breaker conditions (height, period, angle of approach and type), local winds, longshore currents, foreshore slope, width of the surf zone and rip currents were made daily during the period under consideration. The longshore current was determined by observing the direction and measuring the distance parallel to shore that a dye packet injected into the surf zone traveled in one minute. Appendix A provides the set of instructions followed during the observations.

The data used for this study cover a period from May 1972 to September 1975 and refer to stations: 5703, 5706, 5707, 5713, 5714 and 5715, located within the confines of Point Mugu Naval Air Station, 60 miles northwest of Los Angeles, California (location 6 in Figure 5).

From these stations, 4,632 observations were considered of which only 647 data points were used in the analysis. The following criteria were discussed to eliminate observations which were not consistent with the application of the theory or were simply erroneous:

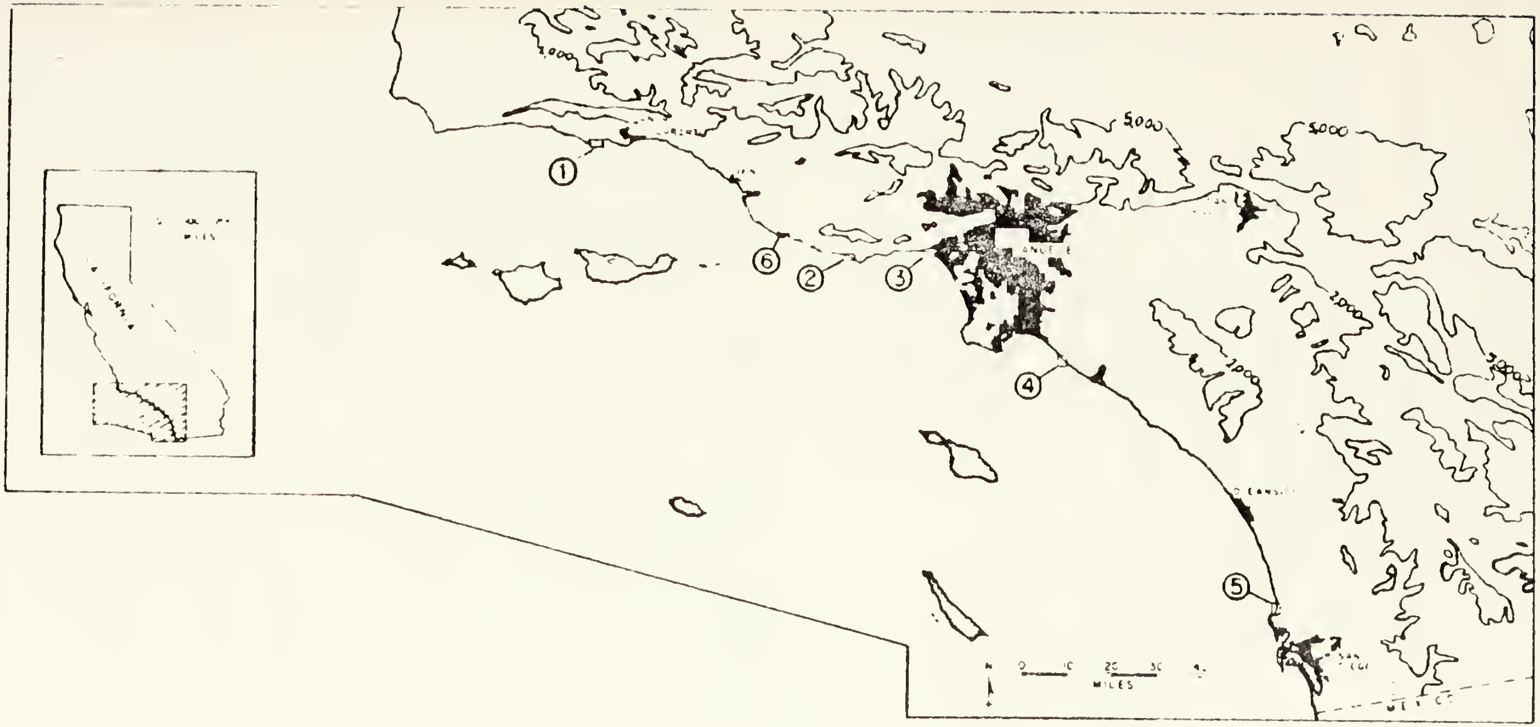


Figure 5. LOCATION MAPS

Location of Southern California coast at which observations were performed. 1 = Goleta Point; 2 = Trancas Beach; 3 = Santa Monica; 4 = Huntington Beach; 5 = La Jolla; 6 = LEO Stations. (Ingle, 1966)

1. Rip Currents

It was beyond the scope of this study to account for any modifying effect of the longshore current system by rip currents. Hence, all observations noting the presence of a rip current were systematically deleted.

2. Angle of Wave Approach

The theory used in the derivation of the formulas employed in this study assumes that the angle between the direction of wave approach and the depth contour must be different from 90 degrees in order for a longshore current to be generated. Hence, all observations in which the wave direction was reported as being perpendicular to the shoreline were neglected.

3. Wind

Shepard and Inman (1950) suggested the importance of the wind in generating longshore current; they also indicate that it is difficult to separate the wind generated current contribution from the current generated by the waves. Thus, observations where the wind speed was reported as being greater than ten miles per hours were not considered.

4. Foreshore Slope

Observations where the foreshore slope was reported as being greater than ten degrees were neglected since such large values are not consistent with what is usually observed on the beaches under consideration.

5. Wave Period

Arbitrarily, to keep the study restricted to sea and swell of relatively short period, all observations where the

wave period was reported as greater than 20 seconds were neglected.

### 6. Doubtful Data

All observations in which the reported data were considered to be incorrect due to either mistakes of the observer or the typist, such as longshore currents in excess of six feet per second, direction of approach greater than 180 degrees, distance of dye injection greater than 600 feet, etc., were systematically rejected.

## B. SOURCES OF ERROR

Considering the interest in longshore currents, it is somewhat surprising that there are relatively few sets of adequate field measurements of longshore currents and the simultaneous wave parameters in the surf zone. After a search of the literature it was concluded that little has been achieved for devising electronic equipment designed for gathering longshore current and associated wave information on a routine basis. Hence, as in the case of the LEO data, most of the observations must rely on the good judgment and personal abilities of the observers. This introduces a subjectivity factor which ultimately affects the final results.

### 1. Breaker Angle

Galvin and Nelson (1967) suggested that the variable most difficult to measure with necessary accuracy is the angle of wave approach or wave direction. Galvin and Savage (1966) suggested that when using a visual compass referenced

to a baseline to measure the breaker angle, the errors may easily be ± two degrees, leading to a relative error which is very large for small breaker angles but which decreases as the breaker angle increases.

In the LEO observations a protractor was used for determining the breaker angle as shown in Appendix A. This system is completely visual using the unaided eye to establish the perpendicular to the shore and introduces a greater human factor. Hence, it is a good assumption to attach an accuracy less than that suggested by Galvin and Savage to such measurements.

### 2. Beach Slope and Surf Zone Width

In describing longshore currents which flow within the surf zone, accurate knowledge is required of the beach profile including both the beach slope and the width of the surf zone. The approach used for the LEO data was to assume a plane beach inside the breaker line. The nearshore subaqueous slope was computed using the observed surf zone width and the observed breaker height. Thus an uncertainty factor for the beach was introduced. Referring to LEO observation instructions in Appendix A, the observation of the surf zone width "... is based upon the judgment of the observer; man-made or natural features in the surf (e.g., a pier) may aid in this observation." Again, a subjectivity factor is involved.

The observed foreshore slope could not be used because it proved to be unrepresentative of the beach profile within

the surf zone. The calculated water depths inside the breaker line using this slope were systematically greater than the calculated breaker depths.

### 3. Wave Period

Galvin and Nelson (1967) point out that under favorable conditions the wave period can be measured with reasonable consistency in the field by visual observation. Although this parameter was not used directly in the computation it is interesting to notice that their suggestion agrees quite well with the LEO observations since the range of periods found fall into the expected values for the shoreline under consideration.

### 4. Breaker Height

In the LEO program, the breaker height observation is based solely on the judgment of the observer. Known dimensions of natural or man-made features on the shoreline or in the surf zone are used as references for estimating the wave breaking height. Galvin and Savage (1966) suggested a relative error in breaker height measurement of ± 25 percent. They arrived at this figure by comparison of breaker height measurements made with pressure gages, oscillographs and visual observations, although the measurements were not made simultaneously. Hence, in the light of their finding it can be concluded that at least the same error should be expected in the LEO data in which the observations are solely visual.

### 5. Longshore Current

In the LEO program, the current speed was determined by using a dye as a tracking agent. This also adds an uncertainty factor due to the diffusivity characteristic of the dye.

## C. INGLE DATA

A set of 62 field observations made along the Southern California coast (Fig. 5), taken from Ingle (1966) and personal communication, were selected using the same criteria used for selecting the LEO data. Despite the size of Ingle sample, about ten percent the size of the LEO data, its analysis is important since the Ingle observations are more accurate than the LEO observations. Thus, the Ingle results serve as a reference comparison to the results obtained using the LEO data.

The parameters $H_b$, period, $\alpha_b$ and V were taken directly from a summary appendix in Ingle (1966). The beach profiles and the distance shoreward from breaker zone in which the longshore current velocities were recorded also were available. However, the positions of the breaker were not available. Thus, the parameter $h_b$ was obtained from the relationship $h_b = 1.28\ H_b$; and the local depth below still water level, h, and the beach slope were scaled out from the beach profiles presented in the publication.

It should be pointed out that in the Ingle data the breaker heights were measured by sighting on either a graduated pole held at an approximate still-water line and the horizon,

a graduated pole held in the zone of breaking waves, or a piece of cardboard with a slit and graph paper along one edge. Breakers less than 2 feet in height were estimated while standing in the breaker zone. In the LEO program, breaker height observations were based solely on the judgment of the observer on the shore. For measuring the breaker angle, Ingle observers used a Brunton compass while standing in the surf zone, supplemented by sights taken from positions elevated above the beach; LEO observers used a protractor, as shown in Appendix A, with the observer on the beach. For measuring other parameters, both LEO and Ingle observers used essentially the same techniques. It is important to mention, that in the case of the Ingle observations, the beach slope and the position of the observations in the surf zone were better than those of the LEO observations since in the former an ordinate and abscissa arrangement of wooden stakes allowed workers to position themselves in the surf zone; also most people involved in Ingle observations were well trained personnel.

## IV. RESULTS

Equations (3), (5), (6), (12), (15), (16), (21), (22), (23), (25), (26), (27) and (28) presented in the theory section were used with the parameters $H_b$, $\alpha_b$, $x_b$, $x$ and $V$ from LEO and Ingle field observations to solve for the bed shear stress coefficient. The coefficient calculated is based on data at specific locations within the surf zone and not for mean conditions. It should be mentioned that since the coefficient is not determined by direct measurement, it therefore not only reflects bed shear stress, but also any errors and uncertainties in measurement.

The significance of the assumption that the observer measured waves correspond to a clipped Rayleigh distribution when spatially one-third of the waves have broken, used for computing the reference wave height $H_r$, was tested using other assumptions. It might just as logically be argued that the significant wave height might correspond to the point where half the waves have broken, in which case

$$\exp(-H_s^2/\overline{H_r^2} = 0.5 \ .$$

This assumption was used for computing new values for the coefficient. The relative difference between the coefficients thus calculated and the original ones was determined; the variability was found to be of the order of ten percent,

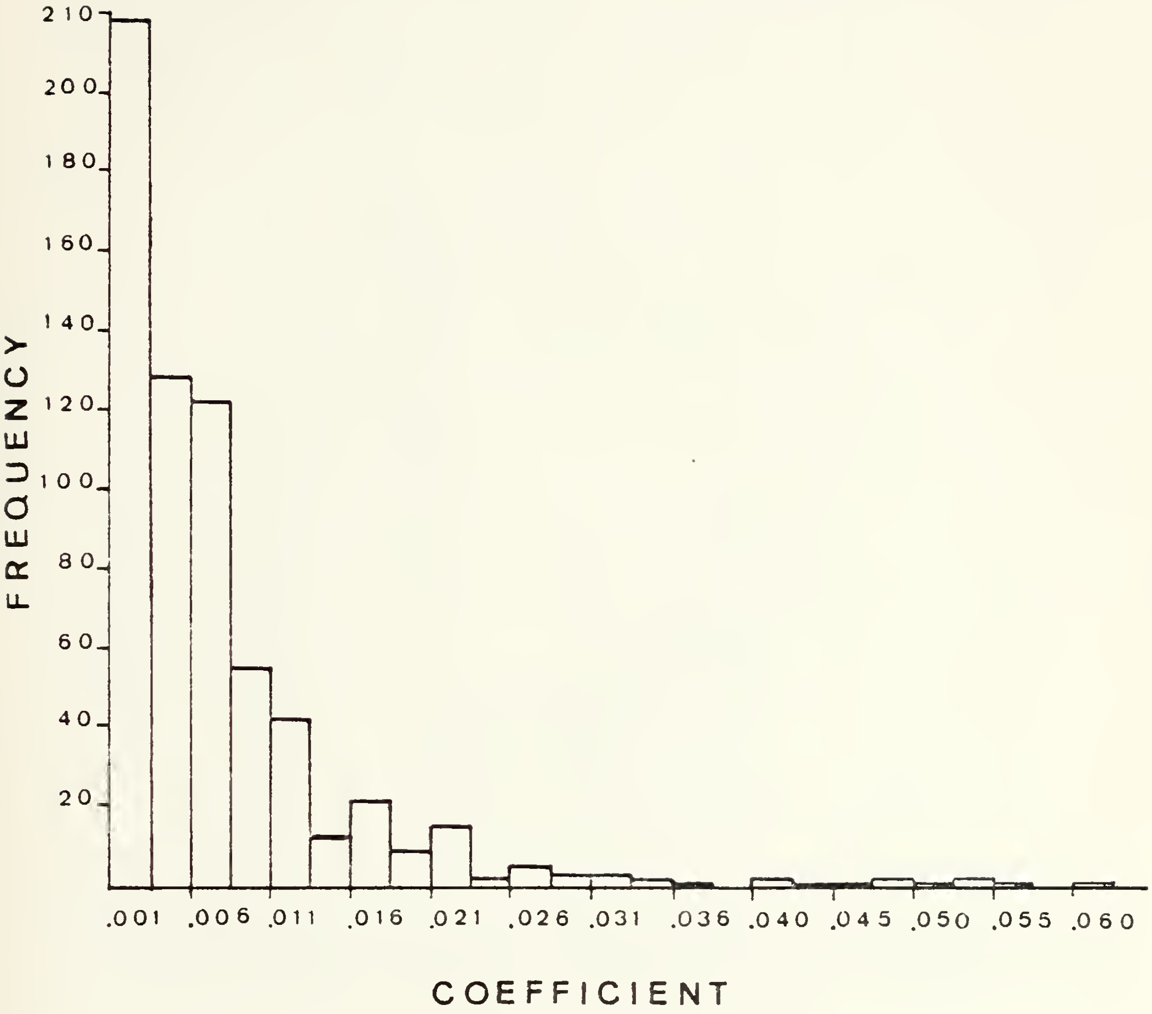
which is relatively low. Hence, it can be said that the bed shear coefficient calculations are not very sensitive to the assumed definition of the significant wave height.

## A. LEO DATA

Figure 6 depicts the frequency distribution of values obtained for the coefficient and selected statistics of the distribution. The variability in coefficient values, as represented by the standard deviation of the distribution, is a measure of the consistency of the calculation of the coefficient from the field observations. The mean of the distribution is 0.008, while the standard deviation is 0.010. This suggests there is a large spreading in the results. However, it should be noticed that more than 90 percent of the calculated values fall between 0.001 and 0.020 and that the distribution has less spread than a Gaussian distribution for the same standard deviation.

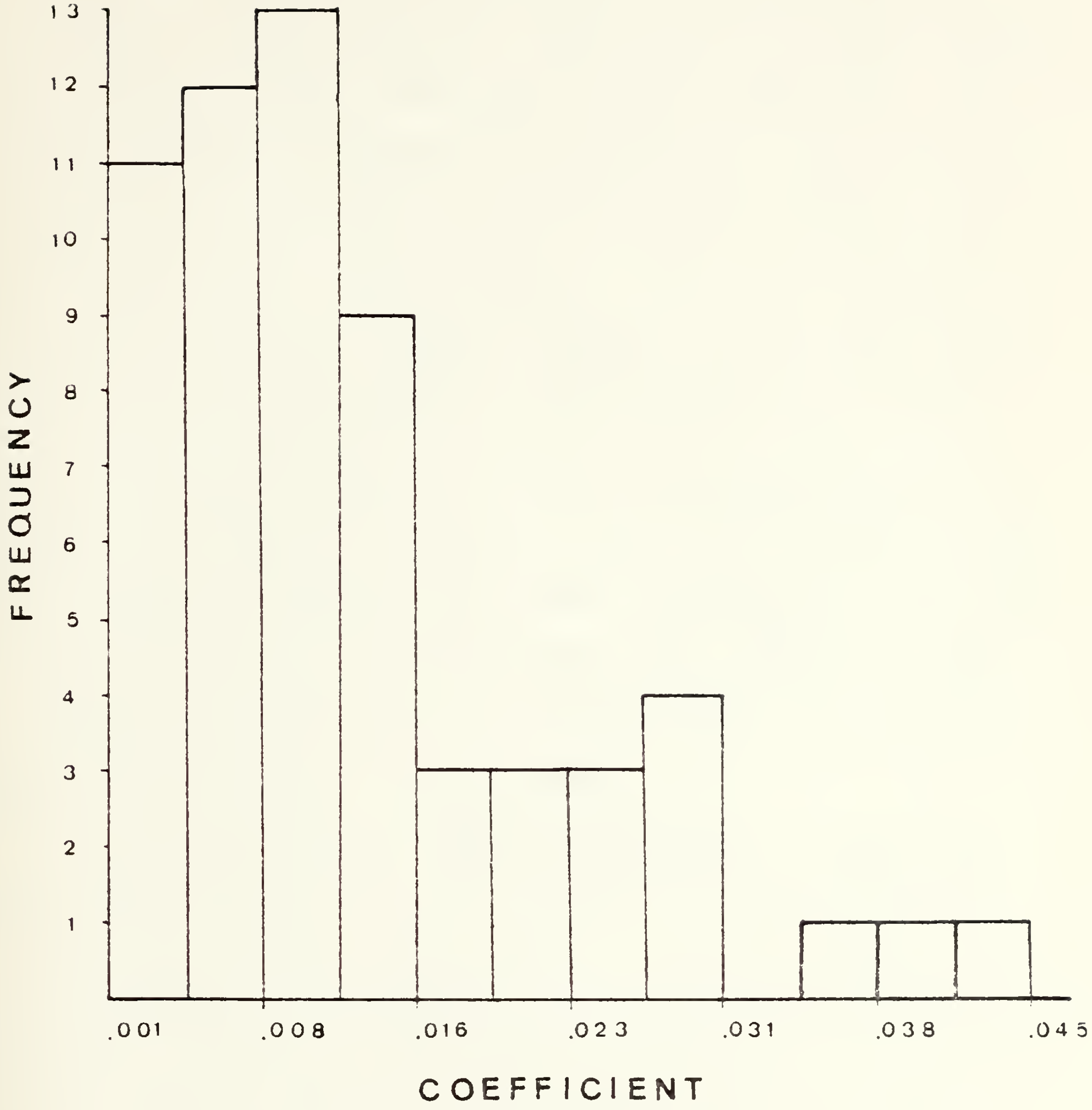
## B. INGLE DATA

Figure 7 depicts the frequency distribution of values obtained for the coefficient and selected statistics of the distribution. The mean of the distribution is 0.014 and the standard deviation 0.011, which suggests again a large spreading in the results. However, 95 percent of values lie between 0.001 and 0.030 and again the distribution has less spread than a Gaussian distribution for the same standard deviation.



| | | | |
|---|---|---|---|
| Mean | 0.008 | Skewness | 3.694 |
| Variance | 0.00009 | Kurtosis | 17.880 |
| Std. Deviation | 0.00972 | Minimum | 0.001 |
| Coef. Variation | 1.19004 | Maximum | 0.080 |

Figure 6. Frequency Distribution of Coefficient Values (LEO Data).



| | | | |
|---|---|---|---|
| Mean | 0.014 | Skewness | 1.584 |
| Variance | 0.0001 | Kurtosis | 2.649 |
| Std. Deviation | 0.0112 | Minimum | 0.001 |
| Coef. Variation | 0.8109 | Maximum | 0.056 |

Figure 7. Frequency Distribution of Coefficient Values (Ingle Data).

## C. COMPARISON OF RESULTS

Despite the difference in sample size between the LEO and Ingle data used for the calculations, some comparisons can be made. A simple way of comparing both sample results is by looking at their mean and standard deviation. The mean and standard deviation corresponding to the values of the coefficient for the Ingle data are both larger than the values obtained for the LEO data. There is a relative difference of 75 percent between the mean of the coefficient values of the two samples; but, the relative difference between the two standard deviations is only ten percent. This says that the distribution values for the coefficient in both samples is nearly the same, although for the Ingle data the values for the coefficient were somewhat larger and with more spread than for LEO data which might be expected for the smaller sample size.

It was stated, when comparing both sets of data the Ingle observations were more accurately taken and more reliable than the LEO observation. Hence, the results obtained with the Ingle data would be expected to be better than the results obtained with the LEO data. To test if there is any statistical difference between the two sets of data, a hypothesis test about the two means obtained was made. The central limit theorem states that, if $\bar{x}$ is the mean of a random sample of size n taken from a population having the mean $\mu$ and the finite variance $\sigma^2$, then

$$z = \frac{\bar{x}-\mu}{\sigma\sqrt{n}}$$

is the value of a random variable whose distribution function approaches that of the standard normal distribution as $n \to \infty$. The variances of the population are unknown, but since both samples are fairly large, it is justifiable to approximate the population variances with the samples variance. Thus, a test statistic can be stated as

$$z = \frac{\bar{x}_I - \bar{x}_L}{\sqrt{\frac{\sigma_I^2}{n_I} + \frac{\sigma_L^2}{n_L}}} .$$

The hypothesis to be tested is the null hypothesis, $\mu_I - \mu_L = 0$, against the alternative hypothesis $\mu_I - \mu_L > 0$, where $\mu$ represents the mean of the population. The evaluation, for the data available, of the z statistic was found to be equal to

$$z = \frac{0.014 - 0.008}{\left(\frac{0.0001}{62} + \frac{0.00009}{647}\right)^{1/2}} = 4.53 .$$

For a level of significance of 0.001 the z statistic for the normal distribution is 3.49. Since the value obtained for the test statistic is larger than the critical value of 3.49, the null hypothesis is rejected with great confidence; and it can be concluded that the difference between both means is statistically significant and cannot be attributed to chance. Therefore the results obtained with Ingle data are better than the results obtained with the LEO data.

## D. CORRELATION WITH INDEPENDENT VARIABLES

Attempts were made to correlate the calculated coefficients with the independent variables, breaker type and wave period, which were recorded in the field but which were not used directly in the computations. Analysis showed nothing conclusive regarding the correlation of the coefficient to the breaker type since the distribution of breaker types among the data was very uneven; the spill/plunge type represented 72 percent of the data and the spilling type 20 percent. Table II shows some selected statistics of the distribution of coefficient values for various breaker types.

TABLE II. Selected Statistics for Distribution of Coefficient According to Breaker Type (LEO Data)

| No. Observation | Spilling 130 | Plunging 27 | Surging 25 | Spill/Plunge 454 |
|---|---|---|---|---|
| Mean | 0.0090 | 0.0075 | 0.0098 | 0.00788 |
| Variance | 0.0001 | 0.00004 | 0.00015 | 0.00009 |
| Std. Dev. | 0.0107 | 0.00609 | 0.01204 | 0.00946 |
| Coef. Var. | 1.1880 | 0.0150 | 0.23321 | 1.20318 |
| Range | 0.0790 | 0.0230 | 0.05300 | 0.07500 |
| Minimum | 0.0010 | 0.0010 | 0.00100 | 0.00100 |
| Maximum | 0.0800 | 0.0240 | 0.05400 | 0.07600 |
| Skewness | 3.720 | 1.3971 | 2.90456 | 3.74730 |
| Kurtosis | 17.60 | 1.0553 | 7.38989 | 18.8567 |

A simple linear regression between the calculated coefficients and the observed period gave the selected statistics of Table III.

TABLE III. Correlation of Coefficient with Wave Period Statistics

| | LEO Data | INGLE Data |
|---|---|---|
| Correlation (R) | - .05370 | - .08479 |
| Std. error of estimate | .00970 | .01120 |
| R squared | .00288 | .00719 |
| Significance | .08574 | .25617 |
| Intercept | .01088 | .01892 |
| Slope | - .00023 | - .00042 |

The negative sign of the correlation coefficient indicates that there is an inverse relationship; that is, the value of the coefficient tends to become smaller as the period increases. However, this relationship is very weak as indicated by the absolute value of R which in both cases is much smaller than one. This result is not surprising since waves in shallow water become non-dispersive or invariant of period.

# V. CONCLUSIONS

An analytical solution for the bed shear stress coefficient was derived using the concept of radiation stress.

The best theory for calculating the variation of wave energy and longshore current, and the resulting bed shear stress coefficient, was to use the truncated Rayleigh p.d.f. for the statistical description of the wave field inside the surf zone. A sinusoidal approximation of the waves was used to calculate the wave set-up. Calculations of the coefficient were made by using suitable sets of data obtained during the LEO observation program and Ingle (1966) observations along the Southern California coast.

Variability in the results obtained for the coefficient values were expected due to subjectivity and uncertainties in the techniques used in the data collection. This is the first test of the bed shear stress coefficient using fairly large setsof field measurements within the surf zone. Even with the uncertainties involved, the analysis resulted in a fairly good agreement between the mean of the calculated coefficients in this work and the values obtained by various investigators for the bed shear stress coefficient for different test conditions and outside the surf zone.

It was shown that the dependence of the coefficient on the wave period is negligible, in agreement with the assumption that waves inside the surf zone are non-dispersive or

period invariant. Since one of the biggest differences between Pacific and Atlantic coast waves is the period, it may be concluded that the calculated coefficient is not ocean dependent.

Since it was initially concluded that Ingle's data was of higher quality than the LEO data, it is assumed the coefficient values using Ingle's data is therefore more reliable. In any event, the mean value of the two data sets are the same to two significant decimal places. Therefore, it is concluded that a reasonable value for the bed shear stress coefficient within the surf zone is 0.01.

# APPENDIX A

# LITTORAL ENVIRONMENT OBSERVATIONS

CERC Form No. 113-72-8 Mar 72 has been designed for keypunching onto computer cards directly (small numbers above each box represent card column numbers). It is recommended a pencil be used. All data should be recorded carefully and legibly. Errors should be corrected by first erasing erroneous data as write overs usually produce illegible data. Make remarks as necessary on the form but record only data in the boxes provided. All observations must be made at the same point on the beach every time (in front of the reference pole).

STATION IDENTIFICATION:
Each site in the "Littoral Environment Observation" study has been assigned a numerical code consisting of 5 digits. The first two digits define the state or territory in which the site is located and the remaining 3 digits define the particular beach or park within the state or territory. A space has also been provided to write in the name of the particular beach or park at which the observation is taking place.

DATE:
Indicate in the spaces provided the year, month and day on which an observation is made.

TIME:
Indicate the time at which the observations are being made. The 24-hour system of recording time has been selected in order to eliminate any confusion between AM and PM. The hour "00" refers to midnight, "07" to 7:00 AM, "13" to 1:00 PM, etc.

SURF OBSERVATIONS:

a. Wave Period - Record the time in seconds for eleven (11) wave "crests" to pass some stationary point. Eleven "crests" will include ten complete waves (crests and troughs). The first (1) "Crest" selected for observation is recorded as time zero and the eleventh (11) "crest" will be the stop or cut time. Record this time in seconds in the spaces provided.

b. Breaking Wave Height - This observation is based solely on the judgment of the observer. Natural or manmade features on the shoreline or in the surf zone whose dimensions are known may aid in judging the height of a wave. Otherwise the observer's best estimate will be sufficient. Record the breaker height to the nearest tenth of a foot.

c. Breaker Angle - To determine the direction from which the waves are approaching the beach use the protractor on this reverse side of the data form. The 0-180° line should be oriented along the shoreline; use the protractor to site the direction from which the waves are approaching when they are first breaking.

d. Type of Breaking Wave:

Spilling - Spilling occurs when the wave crests becomes unstable at the top and the crest flows down the front face of the wave producing an irregular, foamy water surface. (see figure 1)
Plunging - Plunging occurs when the wave crest curls over the front face of the wave and falls into the base of the wave producing a high splash and much foam (figure 2)
Surging - Surging occurs when the wave crests remains unbroken while the base of the front face of the wave adcanoes up the beach (see figure 3)
Spill/Plunge - A combination of both spilling and plunging occurring simultaneously.

WIND OBSERVATIONS:

a. Wind Speed - A wind meter is provided to each observer and it is recommended that the instructions provided with the meter be followed to obtain wind speed measurements.

b. Wind Direction - After the approximate orientation of the beach with respect to north has been defined the observer can determine the direction "from which" the wind is coming.

FORESHORE SLOPE:
For measurement of the foreshore one must use either the clipboard/inclinometer or the Abney hand level. Observations should be made as close to mid-swash as possible. Using the clipboard/inclinometer place it on the appropriate edge and record the angle where the ball comes to rest. Using the Abney hand level place it on a straight edge and level the bubble; record the indicated angle.

WIDTH OF SURF ZONE:
This observation is based solely on the judgment of the observer. Estimate in feet the distance from the shoreline to the line of the most seaward breakers (not to be confused with white caps).

LONGSHORE CURRENT:

a. Dye Distance - Dye packet should be injected just shoreward of the breakers, if possible. Driftwood or any other floating object should be used if dye is not available. Estimate the distance from the shoreline to point of injection and record this distance in feet.

b. Current Speed - Mark the beach in line with the injected dye and make a second mark to indicate the dye movement after one minute has lapsed. Pace the distance between these marks and record this distance in feet.

c. Current Direction - When looking seaward, if the dye has moved to the left record -1, to the right record +1, and no longshore movement record 0.

RIP CURRENTS:
Rip currents are defined as seaward moving channels of water which return the water that has been piled up along the shore by incoming waves. Rip currents are fed by feeder currents, water moving along the shore (see figure 4). Two currents join and extend out in what is known as the "neck", where the water rushes through the breaker zone in a narrow lane. Beyond the breaker zone the current spreads out and dissipates in what is called the "head". If such rip currents are present estimate their spacing in feet. If no rips are present record 0.

BEACH CUSPS:
Cusps are semicircular or crescent shaped cutouts in the beach face (see figure 5). If such shapes are observed record the distance between the "horns" of the cusps which indicate the spacing. Where the spacing is irregular estimate the average spacing. If no cusps are present record 0.

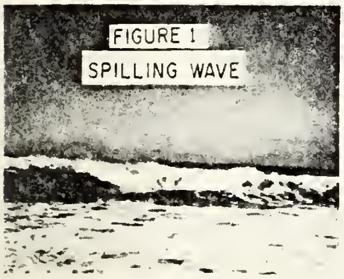
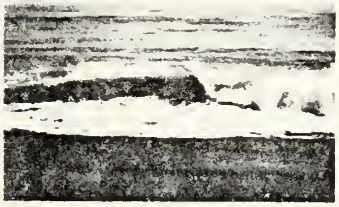


FIGURE 1
SPILLING WAVE

FIGURE 2
PLUNGING WAVE



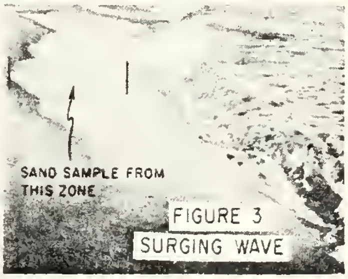


FIGURE 3
SURGING WAVE

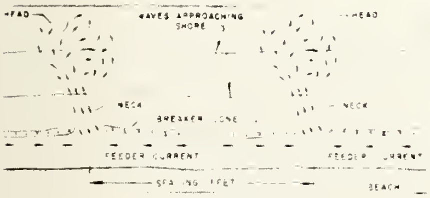


FIGURE 4.

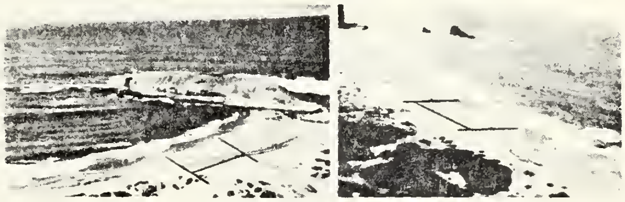
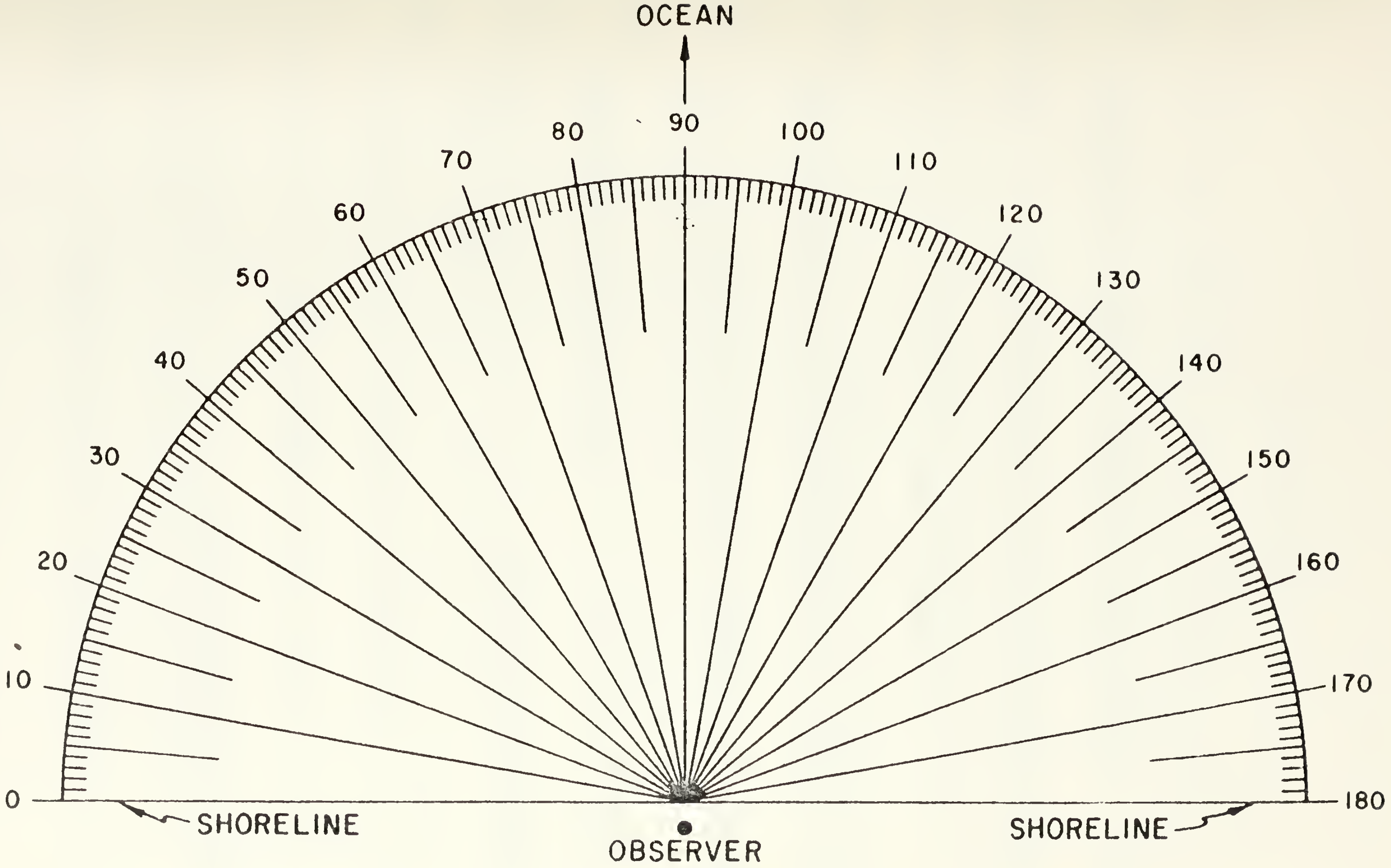


FIGURE 5 BEACH CUSPS



NOTE: If a pier is used for an observation platform: place 0–180 line on the rail parallel to the centerline of the pier, site along the crest of the breaking waves and record the angle observed.

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